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PROJECT SHOAL

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FALLON, NEVADA
OCTOBER 26, 1963

TEST OF DRIBBLE-TYPE STRUCTURES

Holmes & Narver, Inc.

January 1964

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PROJECT SHOAL

TEST OF DRIBBLE-TYPE STRUCTURES

JANUARY 1964

Prepared by:

HOLMES & NARVER, INC.
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TEST OF DRIBBLE-TYPE STRUCTURES

A. INTRODUCTION

1. Objective and Authority

This task was conducted at Project Shoal to observe the response of simulated Dribble area building foundations to ground motions induced by a nuclear detonation.

Official approval for this work is contained in AEC Work Authorization 64-74, dated July 19, 1963. This project was prosecuted in accordance with the Project Shoal Operation Order (Revision 2), Annex A of Appendix B, dated September 30, 1963.

2. Background

The possibility of damage to many privately owned structures near the site of the proposed Salmon Event of Project Dribble, near Purvis, Mississippi, motivated the scheduling of pre-shot and postshot surveys of those structures. The pre-shot survey, which was made in April and May of 1963, included 1) a thorough inspection, 2) photographic and narrative documentation of condition, and 3) recommendations for pre-shot structural bracing, where feasible, of each structure. The postshot survey will document as recorded evidence any changes that may have occurred between surveys.

Many of the buildings near the Dribble site are supported on pedestals of stacked and unbonded concrete blocks, and most of the recommended bracing was intended to improve the lateral stability of this type of foundation. The activities reported herein were devised to test the need for bracing and the effectiveness of the bracing methods prescribed in the pre-shot Dribble survey.

B. PROCEDURE

1. Experiment Design

The residential building nearest to the Salmon Event is situated about 5400 feet, or 1.02 miles, from surface zero (SZ). The residence at that location, which became the prototype for this experiment, is founded on stacked-block piers. Further description of this and all other buildings within a 4.5 mile radius of the Salmon SZ is included in the Pre-shot Damage Report prepared by Holmes & Narver, Inc. (H&N) in May 1963.

R. F. Beers, Inc. predicts that the maximum possible peak ground motions at one mile from the Salmon SZ will be 13 inches per second velocity and 2 g's acceleration; whereas the most probable values for velocity and acceleration will be 50% less at the same distance. Beers, Inc., made no predictions of displacements.

Assuming that the amplitude of peak ground particle velocity provides the best measure of structural damage, two sets of test structures were located at ranges from the Shoal SZ where the peak particle velocities were expected to be 13 and 6.5 in./sec., respectively. Using H&N prediction curves, and considering the expected yield of 12.4 kt at a depth of burial of 1200 feet in a decomposed granite medium, the horizontal ranges from SZ for the two desired velocities were determined to be 4200 feet (Station A) and 6200 feet (Station B).

The prototype is a one-story frame building some 30x40 feet in plan, which is slightly larger than an average Dribble area residence. As a compromise, a test structure size of 24x32 feet was selected as best representing the size of the average Dribble building. The weight of a typical 24x32 foot house was estimated to be 20,000 pounds, with its center of mass located some 5'-9" above the floor.

Because of the primary interest in the foundation, the typical Dribble type block piers were erected for this test, but the superstructure was not simulated. The desired building weight was achieved with a timber framework supporting a "sand box" which, when loaded with 20 pounds per square foot of sand, provided an equivalent weight of 20,000 pounds with its center of mass approximately 5'-9" above the floor.

The test structures were deliberately oriented in positions least favorable for stability. The long axis of each structure was normal to a radius from SZ, and parallel to the long axes of the foundation piers.

2. Construction

At both stations, an area large enough for two test structures was rough graded. One of the test structures at each range was founded on fifteen piers. Each pier consisted of three unbonded 8x8x16 hollow concrete blocks. This structure is referred to as the "unbraced" structure. The other structure at each range was similar except for a supplemental timber pier adjacent to each of the fifteen concrete block piers, as prescribed in H&N Pre-shot Damage Report for Project Dribble. This is referred to as the "braced" structure. A tarpaulin was placed over the fill in the sand box to protect it from the weather. The construction drawings, including a site plan, are shown as Figures 23 and 24. Figures 1 through 6 are pre-shot photographs of the structures.

3. Instrumentation

No instrumentation was included specifically for this project. However, the test stations were bracketed by strong-motion installations of the United States Coast & Geodetic Survey (USC&GS) on their "C" line, which bears S 35° W from the Shoal SZ. The USC&GS installation recorded three components of both acceleration and displacement, as a function of time. These gages were located at ranges of 3122 ft., 5686 ft., 11,100 ft., and 18,640 ft., from SZ.

Figure 1   Pre-shot Station A, Rear View Looking North

Figure 2   Pre-shot Station A Unbraced, Rear-Center Pier

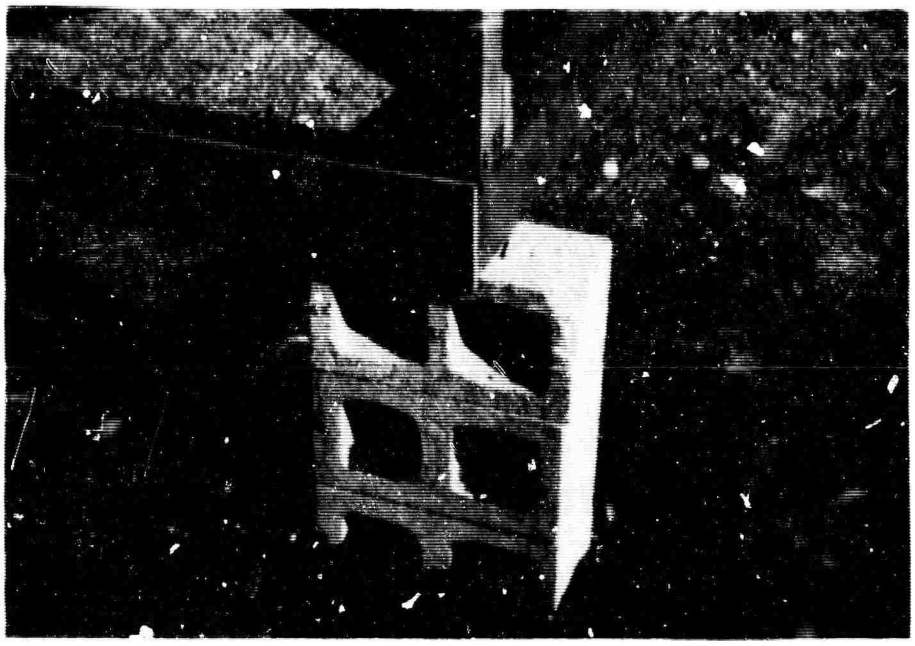


Figure 3 Pre-shot Station A Unbraced Rear-Right Pier

Figure 4 Pre-shot Station A Braced, Rear-Center Pier

Figure 5   Pre-shot Station A Braced, Center-Center Pier

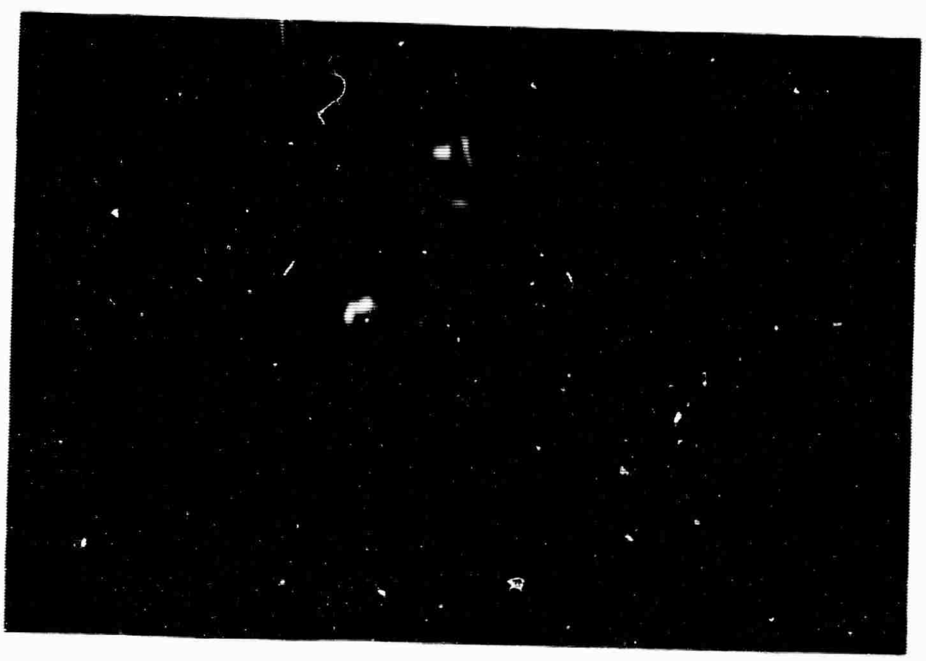


Figure 6   Pre-shot Station A, Center Roof Connection

C. RESULTS

1. Ground Motion Measurements

Ground motion data, as reported by the USC&GS from records of its "C" line stations, are shown in Table 2. The asterisks denote columns containing the recorded data; figures in the other columns of Table 2 were computed from the recorded information, assuming simple harmonic motion of the ground. This is an acceptable assumption when working with displacement records, but the acceleration excursions tend more toward the triangular than the sinusoidal, and the 0.78 factor, applied in the last column to the velocity computed from acceleration, corrects for the difference in area beneath a half-cycle sine wave and the area beneath a right isosceles triangle inscribed in the same half-cycle sine wave.

Figures 7, 8, and 9 are plots of the information tabulated in Table 2. From those plots, the motions at the 4200 ft. (Station A) and 6200 ft. (Station B) ranges of the test stations were obtained by interpolation. Those motions are given in Table 1.

TABLE 1 - GROUND MOTIONS AT TEST STATIONS

HORIZONTAL RANGE	COMPONENT	PEAK ACCELERATION,	PEAK VELOCITY,		PEAK DISPLACEMENT	
		g's	ips	cm./sec.	in.	cm.
4200 ft. Station A	V	1.3	5.9	15	0.6	1.5
	R	0.75	7.9	20	0.6	1.5
	T	0.6	3.5	9	0.24	0.6
6200 ft. Station B	V	0.75	2.9	7.4	0.4	1.0
	R	0.5	3.8	9.6	0.4	1.0
	T	0.4	1.8	4.5	0.2	0.4

TABLE 2 - GROUND MOTIONS, USC&GS "C" LINE

* Station & Range	* Component **	* Accel., Peak g's (cm/sec^2)	* t_a, sec.	* Displ. cm.	* t_d, sec.	f_a, cps.	ω_a (=$2\pi f_a$), rad/sec.	V_a (Accel./ω_a) cm/sec.	f_d, cps	ω_d (=$2\pi f_d$), rad/sec.	V_d (ω_d x displ.) cm/sec.	$V_{average} = \frac{0.78V_a + V_d}{2}$ cm/sec.
0.5C, 3122 Ft	V	1.7 (1666)	0.16	2.6	0.69	6.25	39.2	42.6	1.45	9.1	23.6	28.4
	R	1.0 (980)	0.09	2.7	0.35	11.1	69.7	14.1	2.78	17.45	47.0	29.0
	T	0.98 (960)	0.11	1.1	0.39	9.1	57.1	16.8	2.56	16.08	17.5	15.3
1 C, 5686 Ft	V	0.86 (842)	0.13	0.81	0.81	7.70	48.3	17.4	1.24	7.79	6.3	10.0
	R	0.53 (519)	0.17	0.86	0.56	5.89	37.0	14.1	1.79	11.2[illegible]	9.7	10.4
	T	0.41 (402)	0.12	0.37	0.37	8.34	52.3	7.7	2.70	16.92	6.3	6.2
2 C, 11,100 Ft	V	0.19 (186)	0.10	0.58	0.97	10.0	62.8	2.9	1.03	6.46	3.8	3.0
	R	0.30 (294)	0.12	0.56	0.83	8.3	52.3	5.6	1.21	7.56	4.2	4.3
	T	0.24 (235)	0.10	0.17	1.18	10.0	62.8	3.7	0.85	5.33	0.9	1.9
3.5 C, 18,640 Ft	V	0.049(48)	0.14	N.R.	---	7.15	44.9	1.1	----	----	----	0.9
	R	0.077(75)	0.13	0.30	0.94	7.70	48.3	1.6	1.06	6.65	2.0	1.6
	T	No record	----	0.10	1.18	----	----	----	0.85	5.32	0.5	0.5

*Denotes information provided by USC&GS. Other values were computed from C&GS data.

** LEGEND: V-Vertical R-Horizontal Radial T-Horizontal Transverse

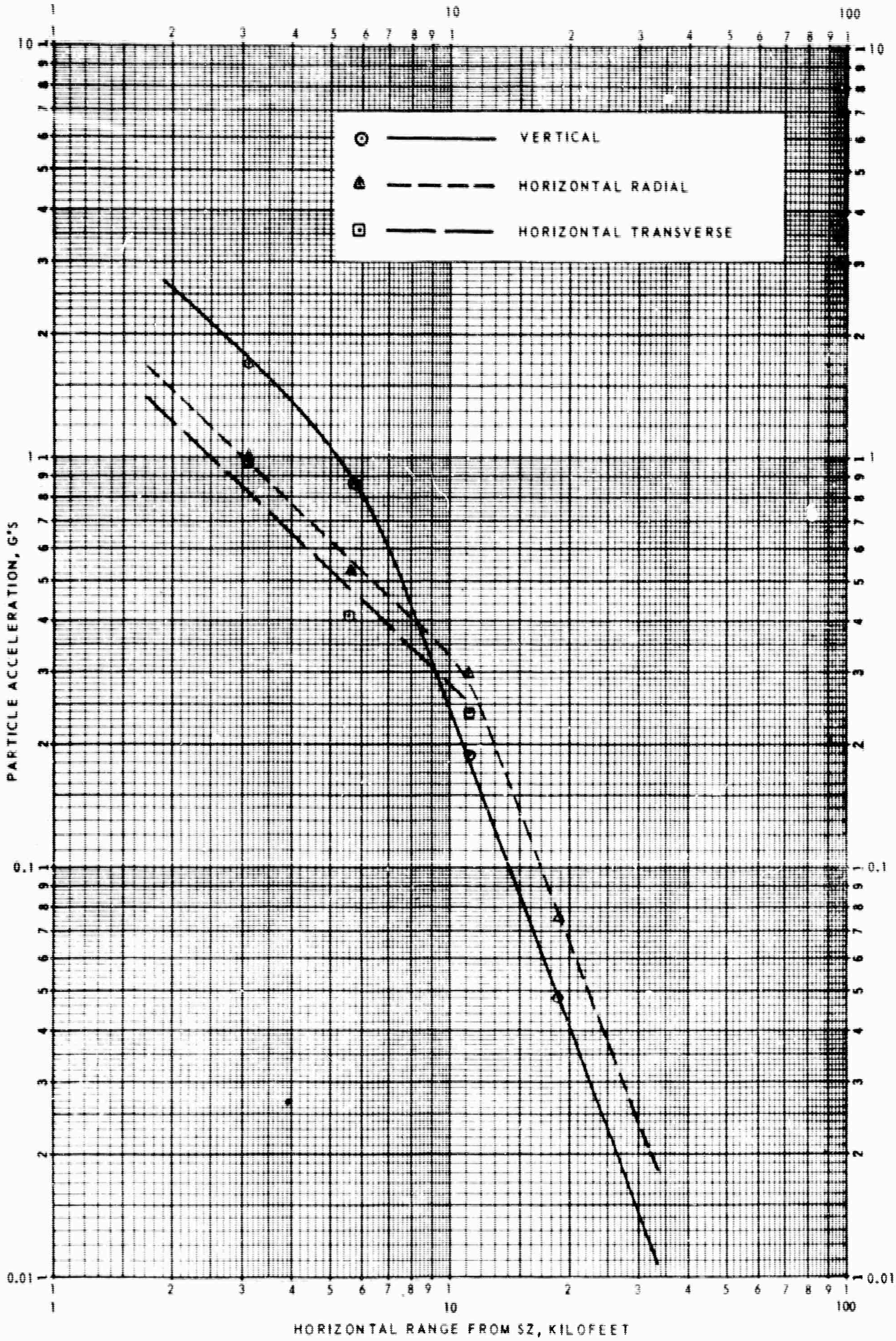


Figure 7 SHOAL - Peak Particle Acceleration vs. Range, USC&GS "C" Line

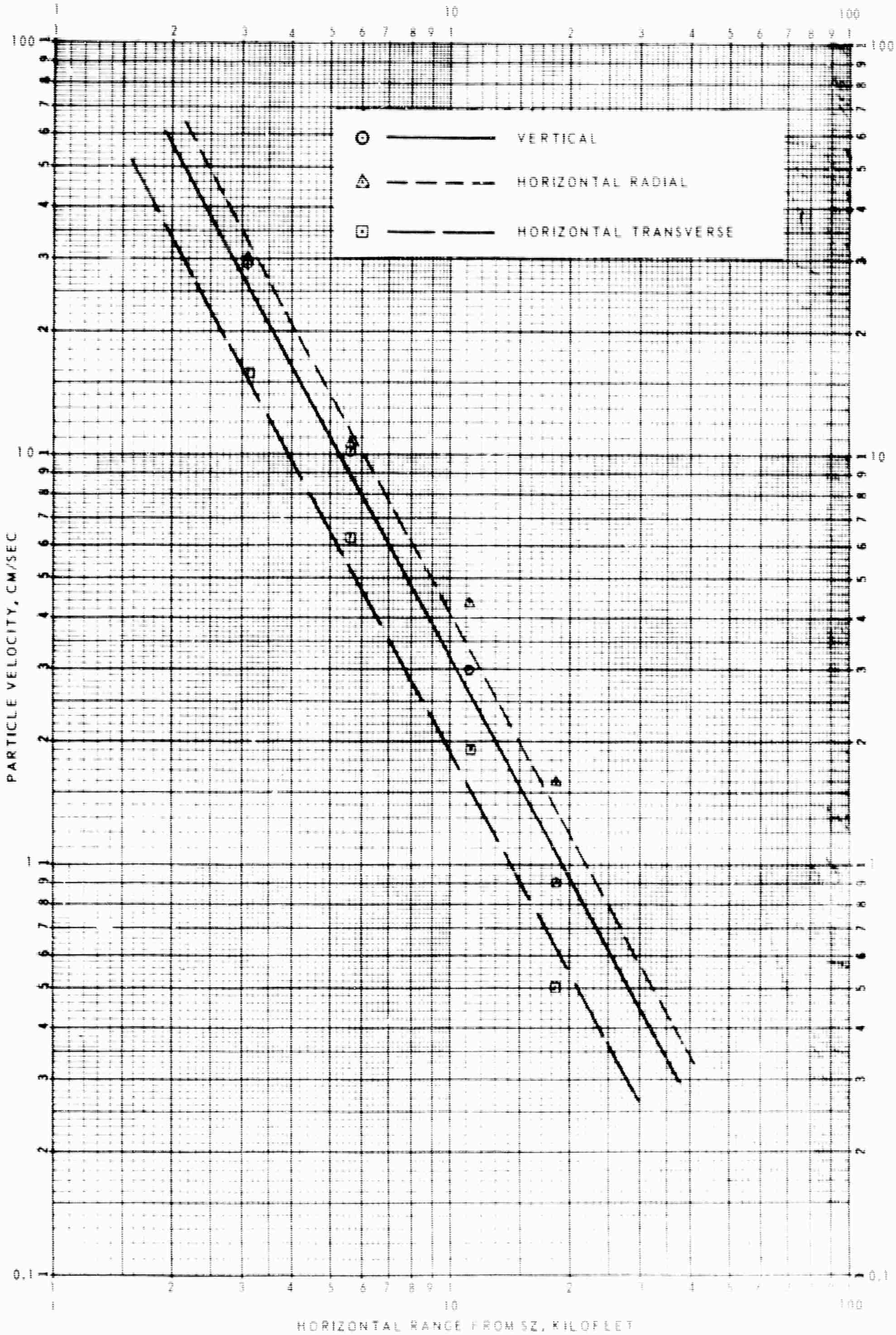


Figure 8 SHOAL - Peak Particle Velocity vs. Range, USC&GS "C" Line

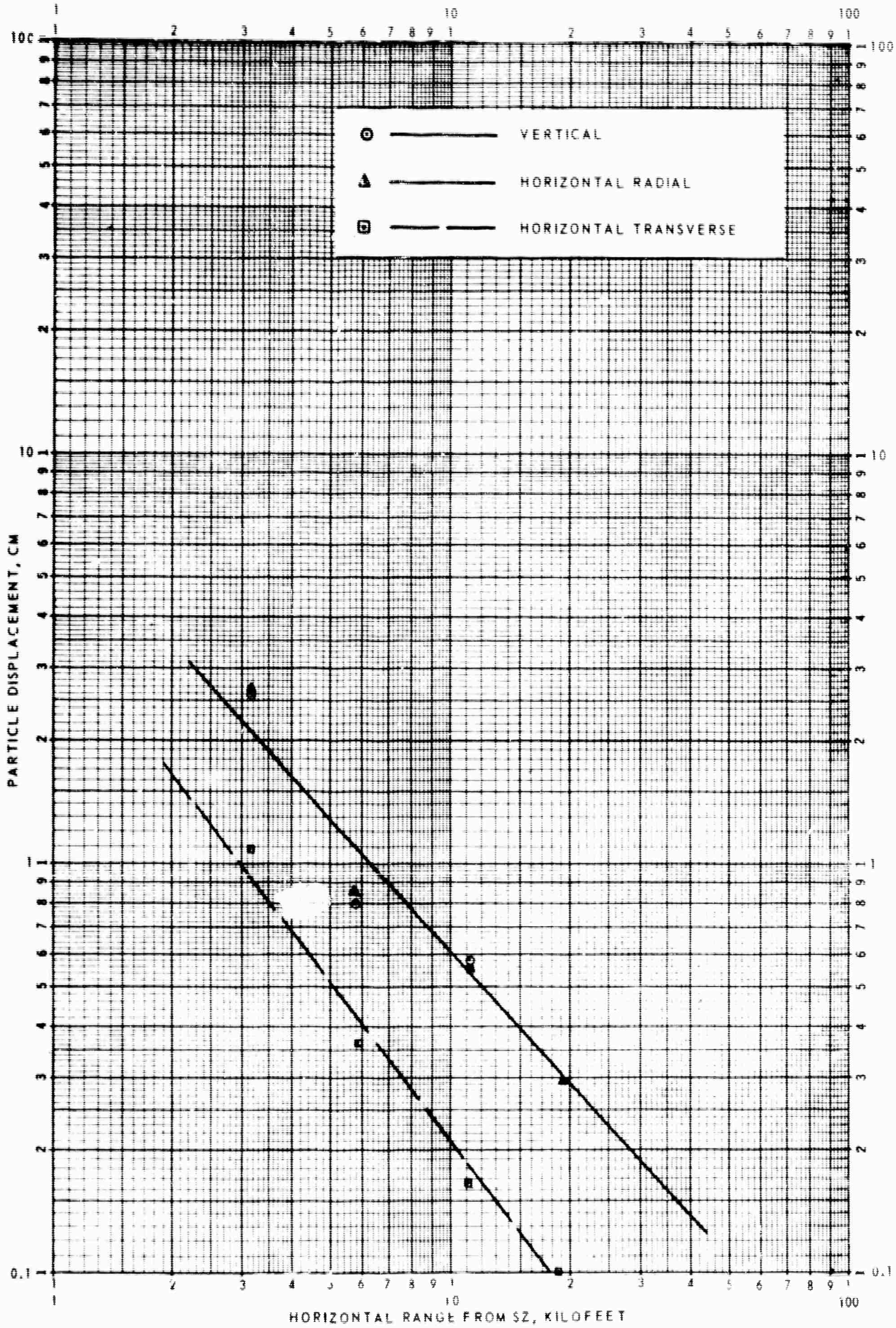


Figure 9 SHOAL - Peak Particle Displacement vs. Range, USC&GS "C" Line

2. Damage

All of the structures remained on their piers. No damage was observed at Station B except for one cracked concrete block under the braced structure (extreme left pier in center row, looking toward SZ). See Figure 10.

At the Station A unbraced building damage was confined to the concrete blocks in the center row of piers. See Figure 11. Four piers in that row were cracked (all but the inner-right pier, facing SZ), and all three of the blocks were cracked in the center and center-left piers; see Figure 16. In the extreme right and extreme left piers of the center row, only the upper blocks were cracked. See Figure 17.

In the unbraced building, there appeared to be a general but slight clockwise turning of upper blocks, with respect to lower ones, of as much as 1/2". The soil around the bottom blocks was undisturbed in the row of piers away from SZ. In the center row there were some 1/8" cracks around a few blocks, and all of the piers in the row toward SZ showed some disturbance of the soil around the bottom blocks.

In the center row, the tops of the piers were displaced away from SZ with respect to the frame, but the piers were still plumb. See Figures 18 and 19. No horizontal differential displacement was observed in the other rows, although the row toward SZ showed a slight deviation from plumb, Figure 20.

Some minor joint distress was observed in the nailed 2x4 diagonals of the superstructure. None of the fastener joints were damaged. The fill appeared to be uniformly distributed in the sand box, but the 1x12 fascia boards were deflected outward as much as 1/4 inch at mid-span.

At Station A in the braced building, the only damage was to the center concrete block pier in the center row. See Figure 21. Only the top block in the pier was cracked.

At all the piers but two (center and center-left piers of center row) the weight was mostly on the timber crib, as evidenced by daylight between the 4x6 sill timbers and the top block of the pier, see Figure 22. The top blocks of both extreme right piers of center row and row toward SZ could be moved by hand, indicating all of the weight at those piers was being supported by the cribs. There was no apparent motion of the frame relative to the pier tops, and no distress in any of the wood cribs.

Figure 10 Postshot Station B Braced, Center-Left Pier

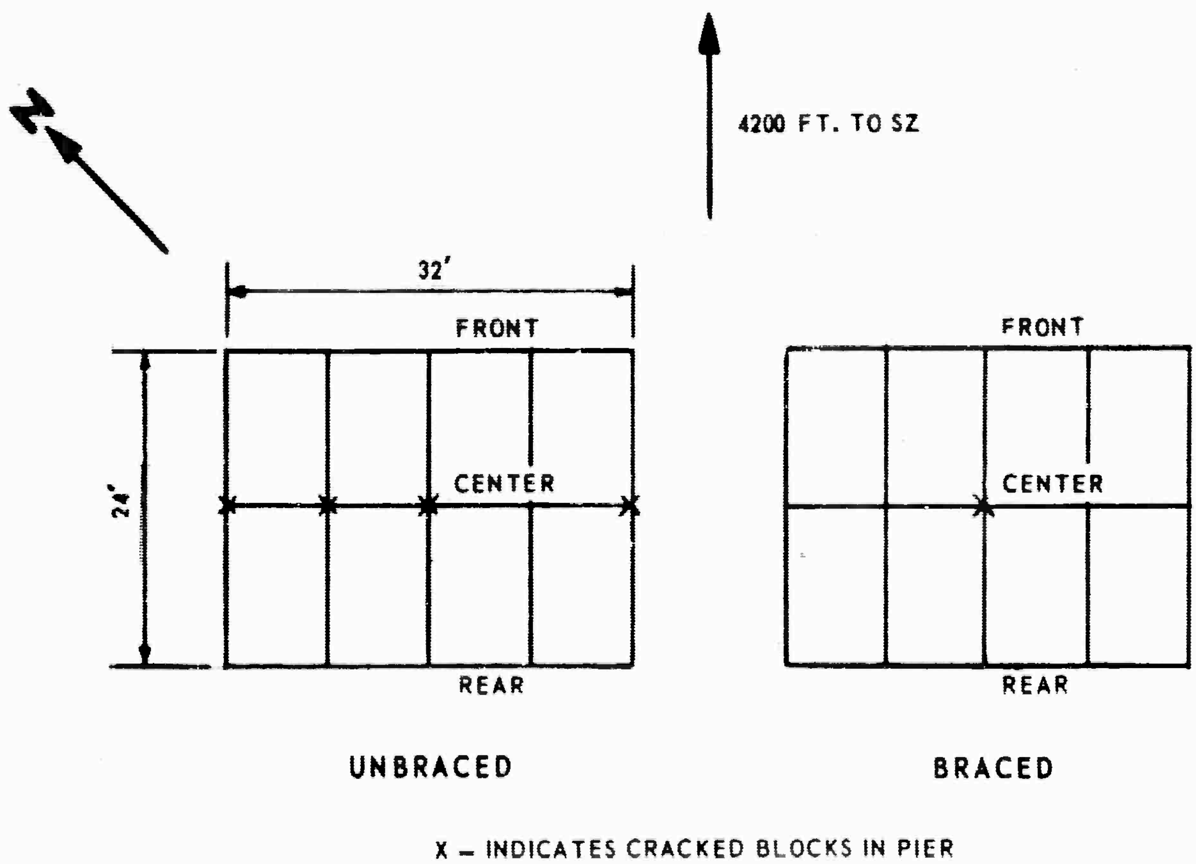


Figure 11 Postshot Station A, Foundation Plan

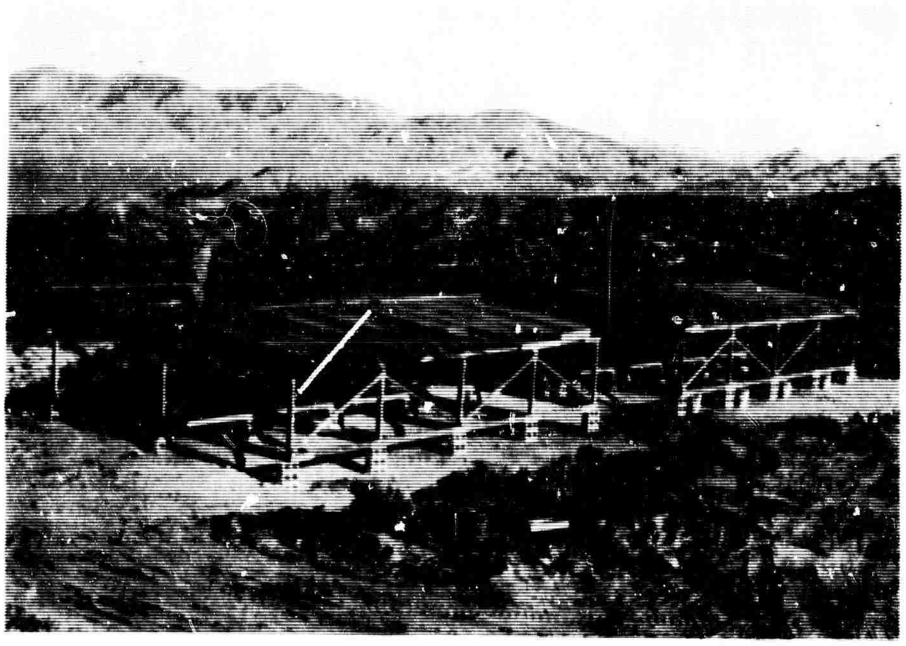


Figure 12 Postshot Station A, Rear View Looking SE, SZ to Left

Figure 13 Postshot Station B, Rear View Looking SE, SZ to Left

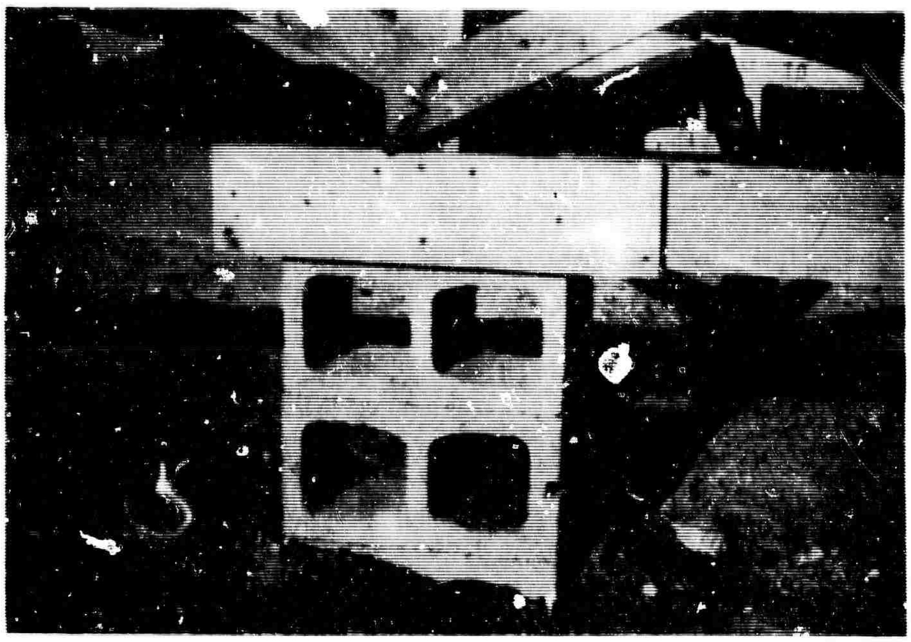


Figure 14 Postshot Station A Unbraced, Rear-Center Pier

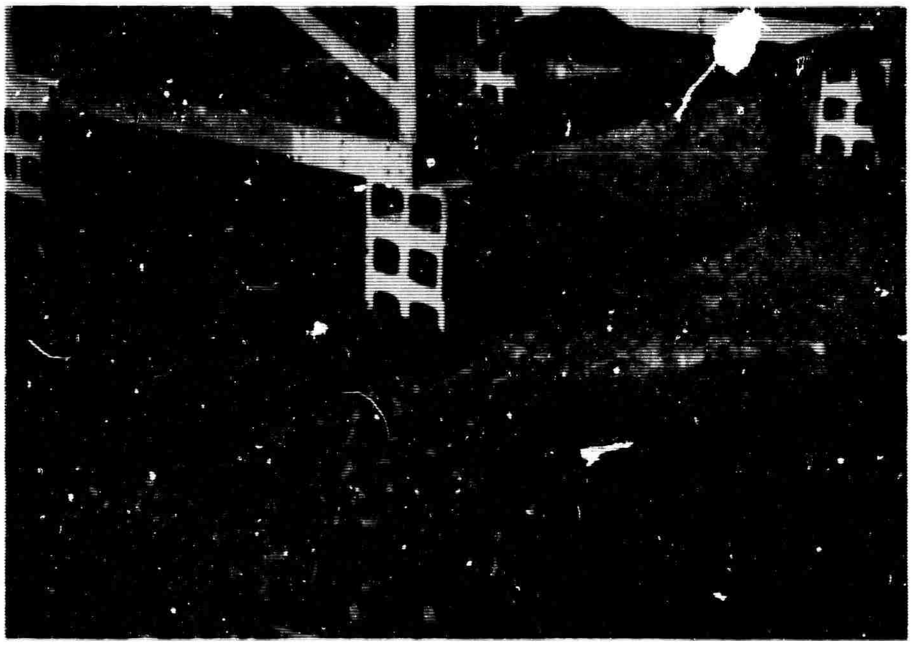


Figure 15 Postshot Station A Unbraced, Rear-Right Pier
Note Crack in Ground, Foreground

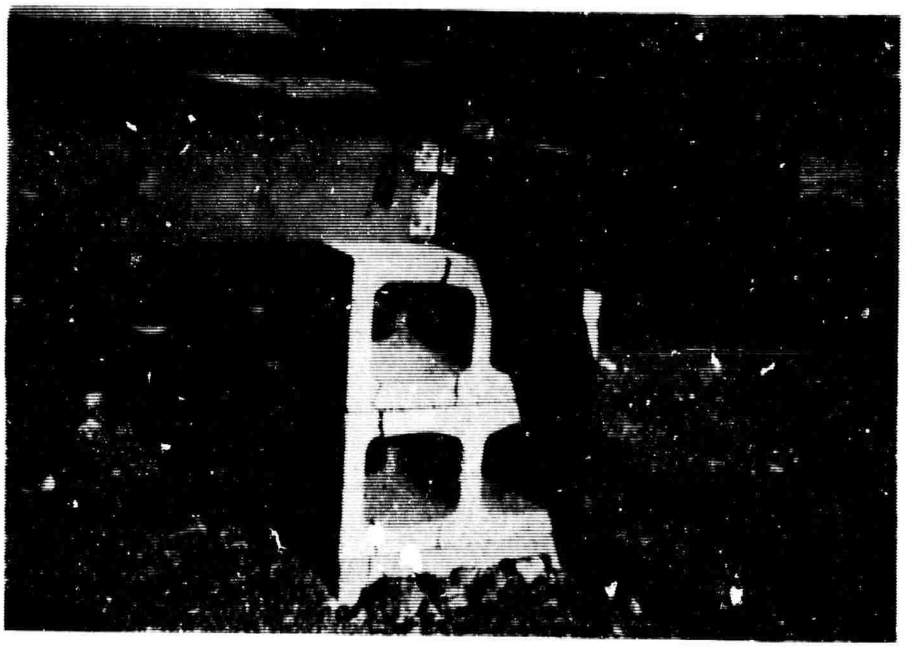


Figure 16 Postshot Station A Unbraced, Center-Center Pier

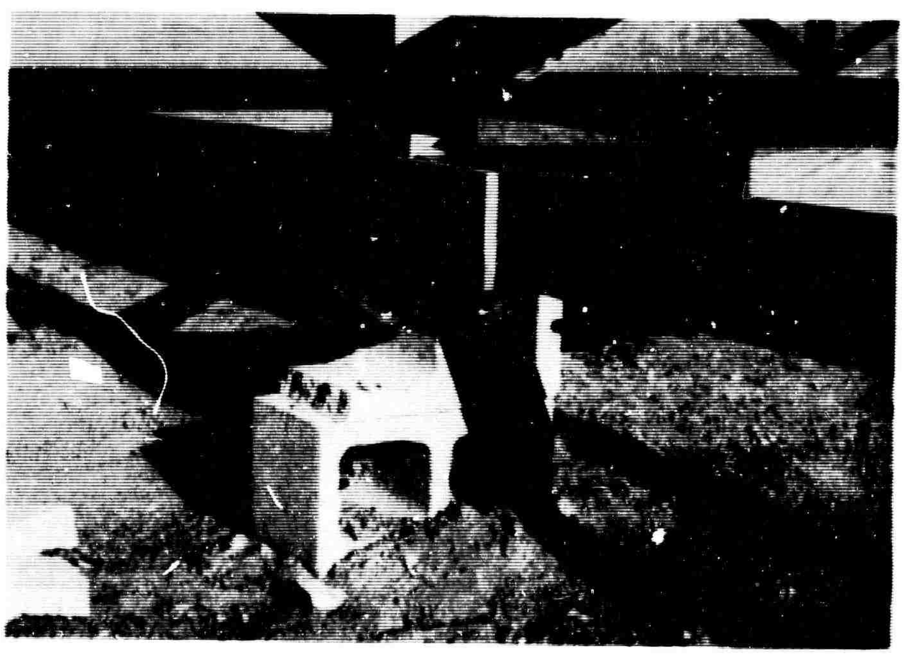


Figure 17 Postshot Station A Unbraced, Center-Left Pier

Figure 18 Postshot Station A Unbraced, Center-Center Pier, SZ to Right

Figure 19 Postshot Station A Unbraced, Center Row of Piers, SZ to Right

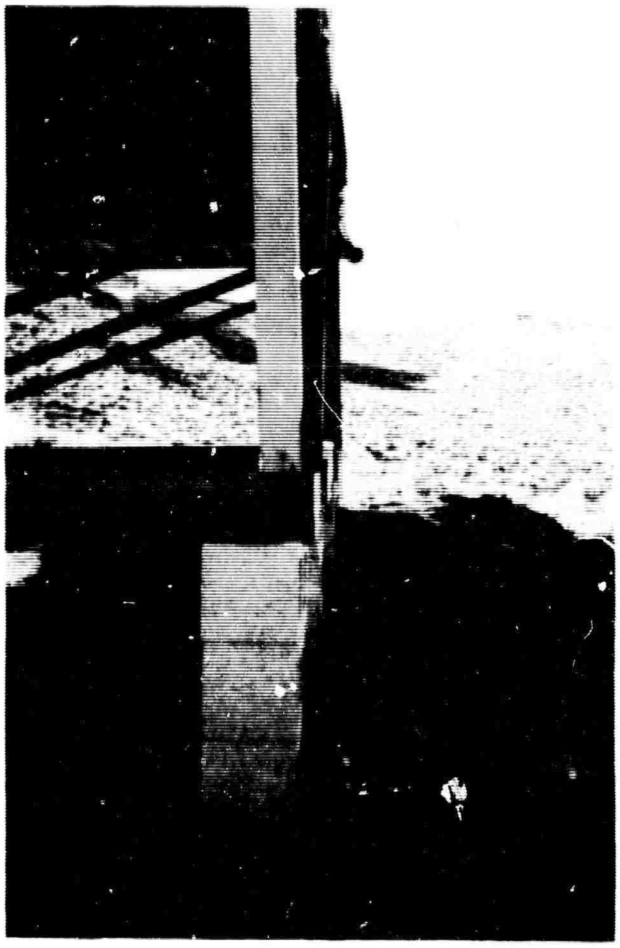


Figure 20

Postshot Station A Unbraced, Front Row of Piers, SZ to Right

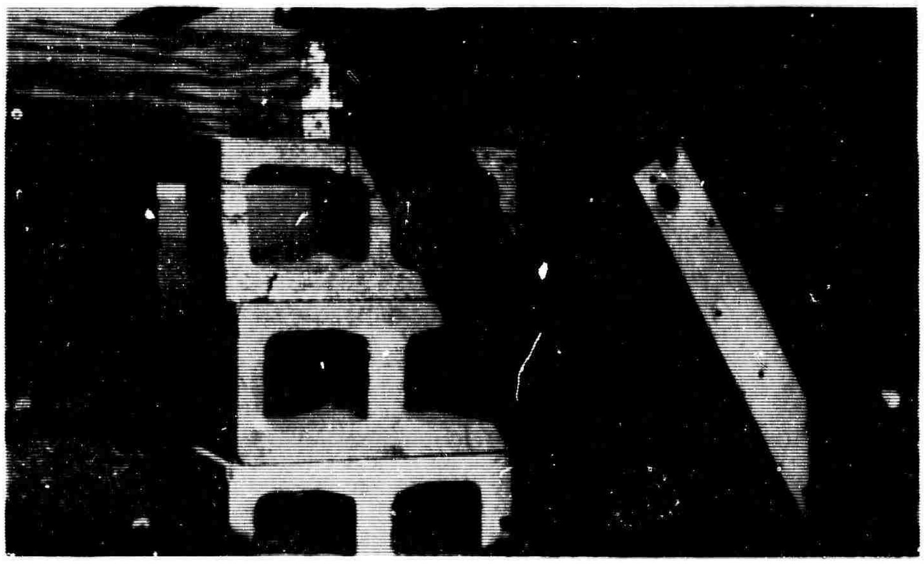


Figure 21 Postshot Station A Braced, Center-Center Pier

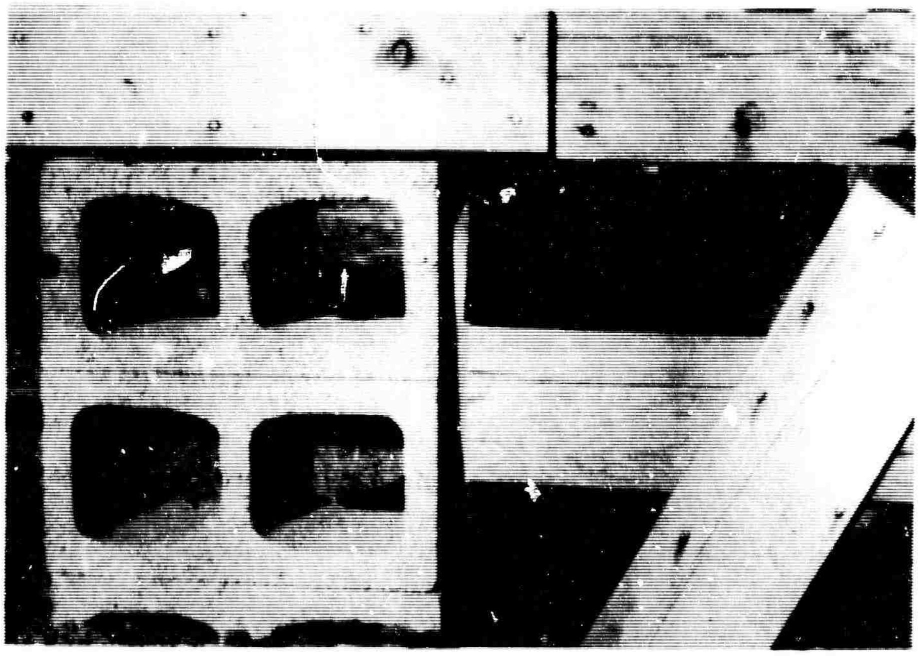


Figure 22 Postshot Station A Braced, Rear-Center Pier

D. DISCUSSION

1. Ground Motions

The interpolated peak motions, as shown in Table 1, are lower by a factor of two than those predicted. This means that the motions recorded at the 4200-foot range (Station A) were comparable to the most probable motions expected one-mile from the Salmon SZ.

The vertical accelerations are seen to be a factor of two higher than either the horizontal-radial or horizontal-transverse peak accelerations. Peak velocities and displacements showed both the vertical and the horizontal-radial components to be about equal, and twice as large as the horizontal-transverse.

2. Damage

This type of foundation (i.e. stacked, unbonded concrete-block piers) appears laterally stable under ground motions of the magnitude observed in this experiment. This stability is not surprising in view of the maximum horizontal-radial ground displacement of 0.6 inch. For the test structure on two-foot high concrete block piers, the natural period of vibration of the structure, computed as an inverted pendulum, was approximately 1 cps. At this frequency the response displacement was also 0.6 inch, which means that the peak displacement of the top of the pier with respect to the bottom (assuming no sliding) was 0.6 inch. This motion of an 8-inch-wide pier should not be sufficient to cause an unstable situation, although causing an increase in maximum soil pressure under the pier of some 40%. In fact, some of the piers were surrounded by soil to a depth as great as six inches (see Figures 2 and 3), which enhanced the lateral stability.

The pier damage appears to have resulted primarily from vertical motion. In the unbraced buildings, the maximum computed static pier reaction (on the two inboard piers of the center row) was 2760 lbs., causing a unit stress of about 100 psi in the vertical "webs" of the blocks. During the motion, this stress could have been momentarily increased, at Station A, to 100 + 1.3 (100) = 230 psi, which is well below the compressive strength of any structural concrete. However, most of the damage is seen to have occurred in the horizontal flanges of the blocks in a manner indicating a shear failure, see Figures 14, 17, and 21.

Further inspection of Figures 14, 17 and 21 shows that the damage is without exception either on one side or the other of the center webs of the pier blocks, but not on both sides. This circumstance indicates uneven bearing of the sill timber on the pier, which is probably to be expected whenever the sill timber is not continuous over a pier. If we assume that the sill is bearing on only half of the top block, the shear stress on flange sections near the webs was over 300 psi, which could result in failure. In fact, a condition of line bearing of the end of a sill timber on the pier might be expected, particularly when the structure slams back down on the pier after the initial upward excursion. If the pier blocks had been stacked with cells vertical instead of horizontal, damage might not have been so extensive because of a reduction in shear and flexural stresses in the blocks.

No meaningful significance is attached to the slight movement of some of the blocks relative to each other, as seen in Figure 21. Such motion is to be expected during the free-fall phase of the motions.

The timber cribs appeared to be effective in reducing pier damage, although they obviously were not required for their original purpose of preventing the building from falling off its piers. To inhibit damage of the type observed at piers where the floor members are discontinuous, a supplementary pier should be placed on all four sides of the existing pier. For instance, a 6x6 timber spiked to the sill timber on all four sides of the existing pier and possibly scabbed together with 1x3's, would provide needed vertical support although providing a lesser amount of horizontal restraint.

The minor damage to a few of the nailed joints in the superstructure, and the foundation distress, indicate that architectural components such as windows and plastered walls probably would have been damaged at these ground motions.

E. CONCLUSIONS

The following conclusions pertinent to the Dribble residence bracing problem have been drawn as a result of this experiment.

1. Peak particle velocities of from 15 to 20 cm./sec. (6 to 8 inches per second) and peak accelerations of up to 1.3 g did not cause lateral collapse of the unbraced pier foundations observed in this project.

2. Such motions did crack the more heavily loaded interior pier blocks.

3. Timber bracing cribs, such as those prescribed in the H&N Pre-shot Damage Report for Dribble, were effective in reducing pier damage. Damage was not, however, completely prevented by the cribs.

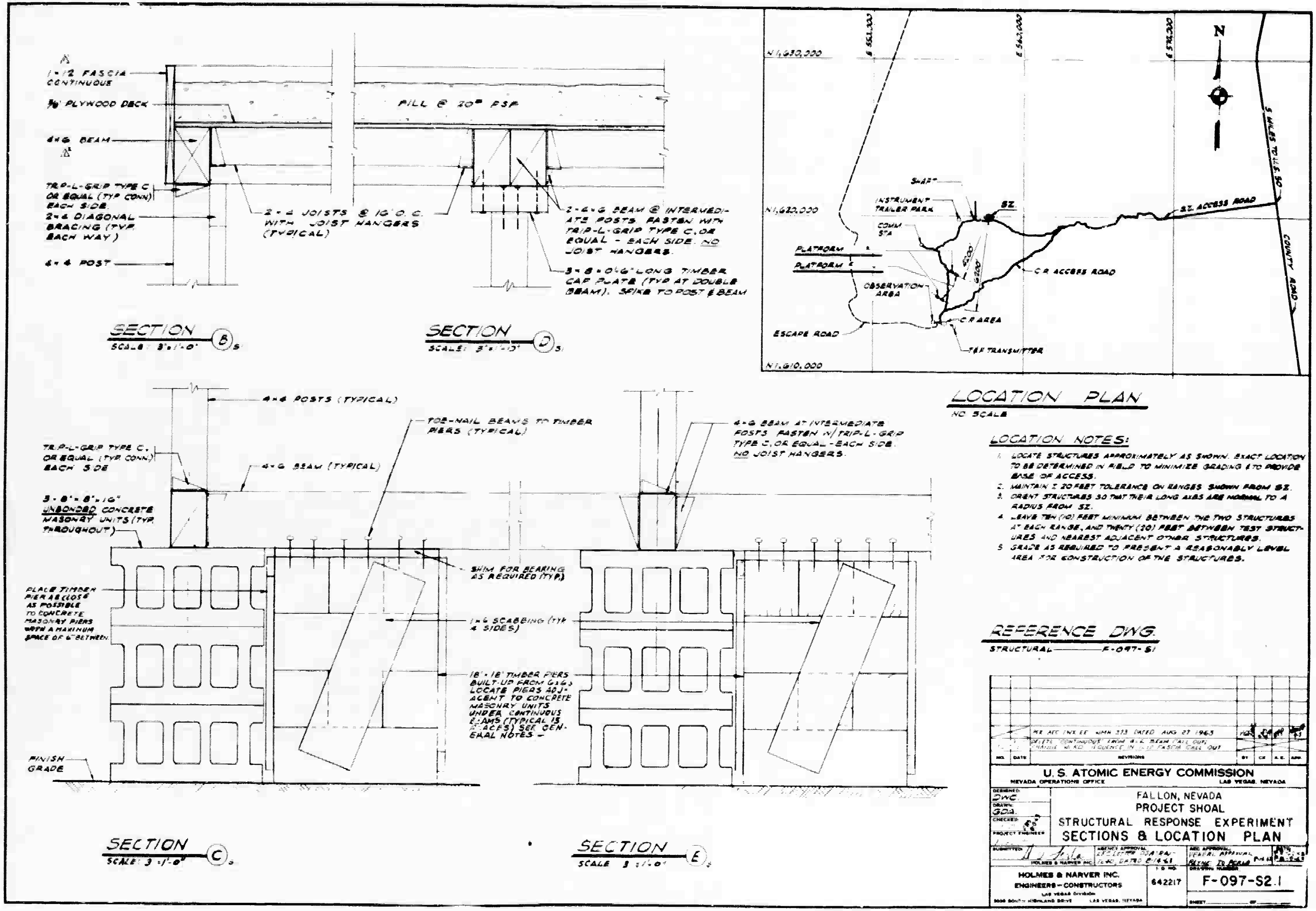
4. No damage to unbraced buildings resulted from peak particle velocities of 9.6 cm./sec. (3.8 inches per second) and peak accelerations of 0.75 g. Peak motions of this magnitude will probably occur as far as 1.6 miles from the Dribble Salmon SZ, and possibly as far as 2.4 miles from SZ.

F. RECOMMENDATIONS

These recommendations apply to bracing foundations of residences in the Dribble area, and modify the recommendations of the Holmes & Narver, Inc., Pre-shot Damage Report, dated May 1963.

To provide increased resistance to vertical motion, it is recommended that where a floor beam timber is discontinuous over a pier, shim and spike a 6x6 timber prop on each side of the pier. If the floor member is continuous over a pier, a 6x6 prop on one side only should suffice. The general recommended configuration at an interior pier is shown in Figure 25.

In general, no piers outside the 2.4 mile radius from the Salmon SZ need be braced.



1×12 FASCIA CONTINUOUS
3/4" PLYWOOD DECK
FILL @ 20# PSF
4×6 BEAM
TRIP-L-GRIP TYPE C OR EQUAL (TYP CONN) EACH SIDE.
2×4 DIAGONAL BRACING (TYP EACH WAY)
4×4 POST
2×4 JOISTS @ 16" O.C. WITH JOIST HANGERS (TYPICAL)
2-4×6 BEAM @ INTERMEDIATE POSTS. FASTEN WITH TRIP-L-GRIP TYPE C, OR EQUAL - EACH SIDE. NO JOIST HANGERS.
3×8×0'-6" LONG TIMBER CAP PLATE (TYP AT DOUBLE BEAM). SPIKE TO POST & BEAM
SECTION B
SCALE: 3"=1'-0"
SECTION D
SCALE: 3"=1'-0"
4×4 POSTS (TYPICAL)
TOE-NAIL BEAMS TO TIMBER PIERS (TYPICAL)
TRIP-L-GRIP TYPE C, OR EQUAL (TYP CONN) EACH SIDE
4×6 BEAM (TYPICAL)
3-8"×8"×16" UNBONDED CONCRETE MASONRY UNITS (TYP THROUGHOUT)
SHIM FOR BEARING AS REQUIRED (TYP.)
1×6 SCABBING (TYP 4 SIDES)
PLACE TIMBER PIER AS CLOSE AS POSSIBLE TO CONCRETE MASONRY PIERS WITH A MAXIMUM SPACE OF 6" BETWEEN
18"×18" TIMBER PIERS BUILT-UP FROM 6×6s. LOCATE PIERS ADJACENT TO CONCRETE MASONRY UNITS UNDER CONTINUOUS BEAMS (TYPICAL IS PLACES) SEE GENERAL NOTES –
FINISH GRADE
4×6 BEAM AT INTERMEDIATE POSTS. FASTEN W/TRIP-L-GRIP TYPE C, OR EQUAL - EACH SIDE. NO JOIST HANGERS.
SECTION C
SCALE: 3"=1'-0"
SECTION E
SCALE: 3"=1'-0"
N
N 1,630,000
N 1,620,000
N 1,610,000
E 551,300
E 563,000
E 574,300
5 MILES TO U.S. 50
COUNTY ROAD
SZ ACCESS ROAD
SZ
SHAFT
INSTRUMENT TRAILER PARK
COMM STA
PLATFORM
PLATFORM
OBSERVATION AREA
C.R. ACCESS ROAD
C.R. AREA
ESCAPE ROAD
T&R TRANSMITTER
LOCATION PLAN
NO SCALE
LOCATION NOTES:
1. LOCATE STRUCTURES APPROXIMATELY AS SHOWN. EXACT LOCATION TO BE DETERMINED IN FIELD TO MINIMIZE GRADING & TO PROVIDE EASE OF ACCESS.
2. MAINTAIN ± 20 FEET TOLERANCE ON RANGES SHOWN FROM SZ.
3. ORIENT STRUCTURES SO THAT THEIR LONG AXES ARE NORMAL TO A RADIUS FROM SZ.
4. LEAVE TEN (10) FEET MINIMUM BETWEEN THE TWO STRUCTURES AT EACH RANGE, AND TWENTY (20) FEET BETWEEN TEST STRUCTURES AND NEAREST ADJACENT OTHER STRUCTURES.
5. GRADE AS REQUIRED TO PRESENT A REASONABLY LEVEL AREA FOR CONSTRUCTION OF THE STRUCTURES.
REFERENCE DWG.
STRUCTURAL F-097-S1
AUG 27 1963
U.S. ATOMIC ENERGY COMMISSION
NEVADA OPERATIONS OFFICE
LAS VEGAS, NEVADA
FALLON, NEVADA
PROJECT SHOAL
STRUCTURAL RESPONSE EXPERIMENT
SECTIONS & LOCATION PLAN
HOLMES & NARVER INC.
ENGINEERS – CONSTRUCTORS
642217
F-097-S2.1

Figure 23

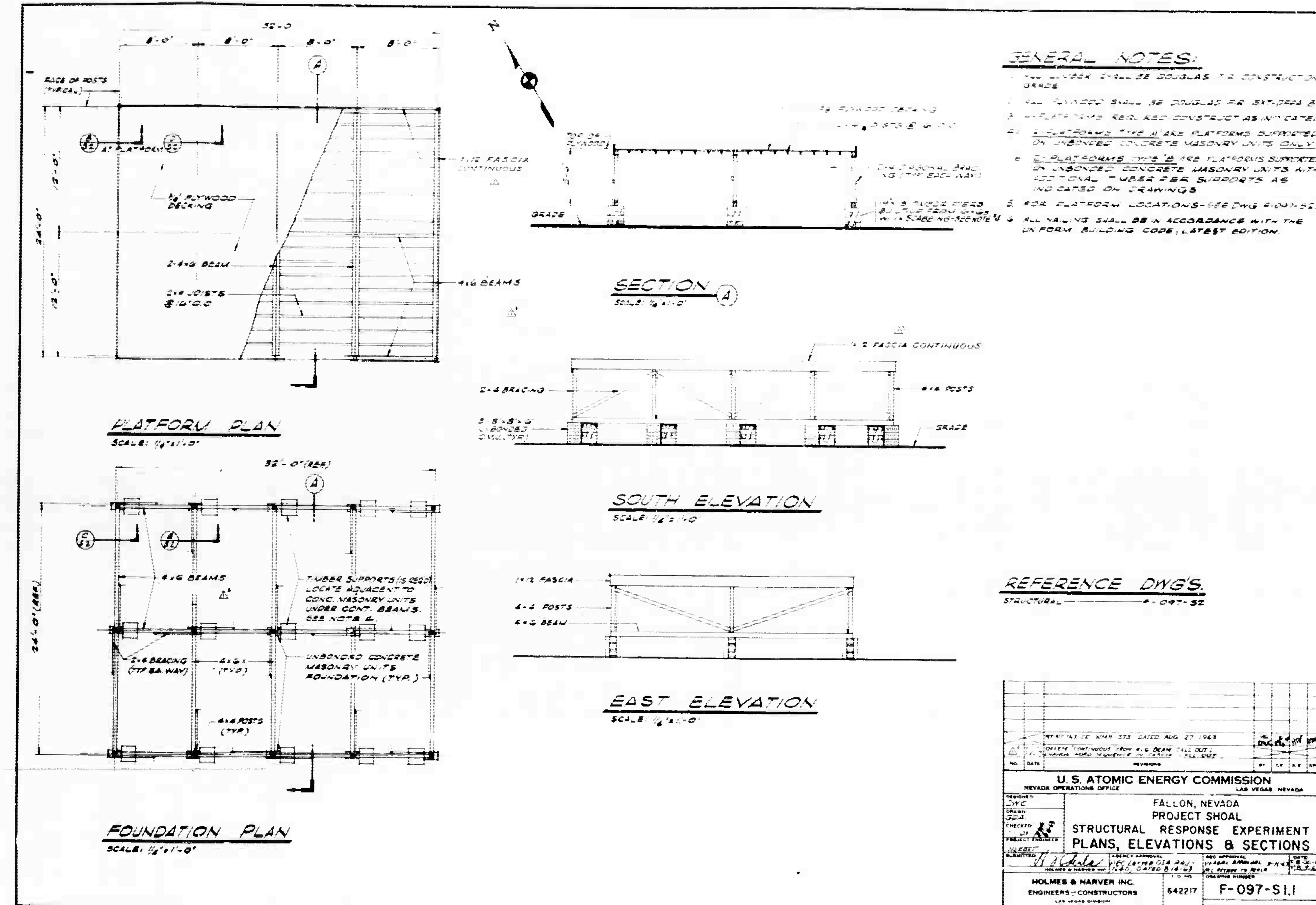

GENERAL NOTES:
ALL PLYWOOD SHALL BE DOUGLAS FIR EXT-DFPA-BC
4. "A" PLATFORMS TYPE "A" ARE PLATFORMS SUPPORTED ON UNBONDED CONCRETE MASONRY UNITS ONLY.
6. "B" PLATFORMS TYPE "B" ARE PLATFORMS SUPPORTED ON UNBONDED CONCRETE MASONRY UNITS WITH ADDITIONAL TIMBER PIER SUPPORTS AS INDICATED ON DRAWINGS.
5. FOR PLATFORM LOCATIONS-SEE DWG F-097-S2.
6. ALL NAILING SHALL BE IN ACCORDANCE WITH THE UNIFORM BUILDING CODE, LATEST EDITION.
PLATFORM PLAN
SCALE: 1/4"=1'-0"
FACE OF POSTS (TYPICAL)
AT PLATFORM
5/8" PLYWOOD DECKING
2-4x6 BEAM
2x4 JOISTS @16" O.C.
1x12 FASCIA CONTINUOUS
4x6 BEAMS
8'-0"
12'-0"
24'-0"
32'-0"
SECTION A
SCALE: 1/4"=1'-0"
TOP OF PLYWOOD
GRADE
SOUTH ELEVATION
SCALE: 1/4"=1'-0"
2x4 BRACING
1x2 FASCIA CONTINUOUS
4x4 POSTS
GRADE
FOUNDATION PLAN
SCALE: 1/4"=1'-0"
32'-0" (REF)
24'-0" (REF)
4x6 BEAMS
TIMBER SUPPORTS (15 REQD) LOCATE ADJACENT TO CONC. MASONRY UNITS UNDER CONT. BEAMS. SEE NOTE 4.
2x4 BRACING (TYP EA. WAY)
4x6's (TYP)
UNBONDED CONCRETE MASONRY UNITS FOUNDATION (TYP.)
4x4 POSTS (TYP)
EAST ELEVATION
SCALE: 1/4"=1'-0"
1x12 FASCIA
4x4 POSTS
4x6 BEAM
REFERENCE DWG'S.
STRUCTURAL — F-097-S2
DELETE "CONTINUOUS" FROM 4x6 BEAM CALL OUT; CHANGE WORD SEQUENCE IN FASCIA CALL OUT
REVISIONS
U.S. ATOMIC ENERGY COMMISSION
NEVADA OPERATIONS OFFICE
LAS VEGAS NEVADA
FALLON, NEVADA
PROJECT SHOAL
STRUCTURAL RESPONSE EXPERIMENT
PLANS, ELEVATIONS & SECTIONS
HOLMES & NARVER INC.
ENGINEERS - CONSTRUCTORS
LAS VEGAS DIVISION
642217
F-097-S1.1

Figure 24

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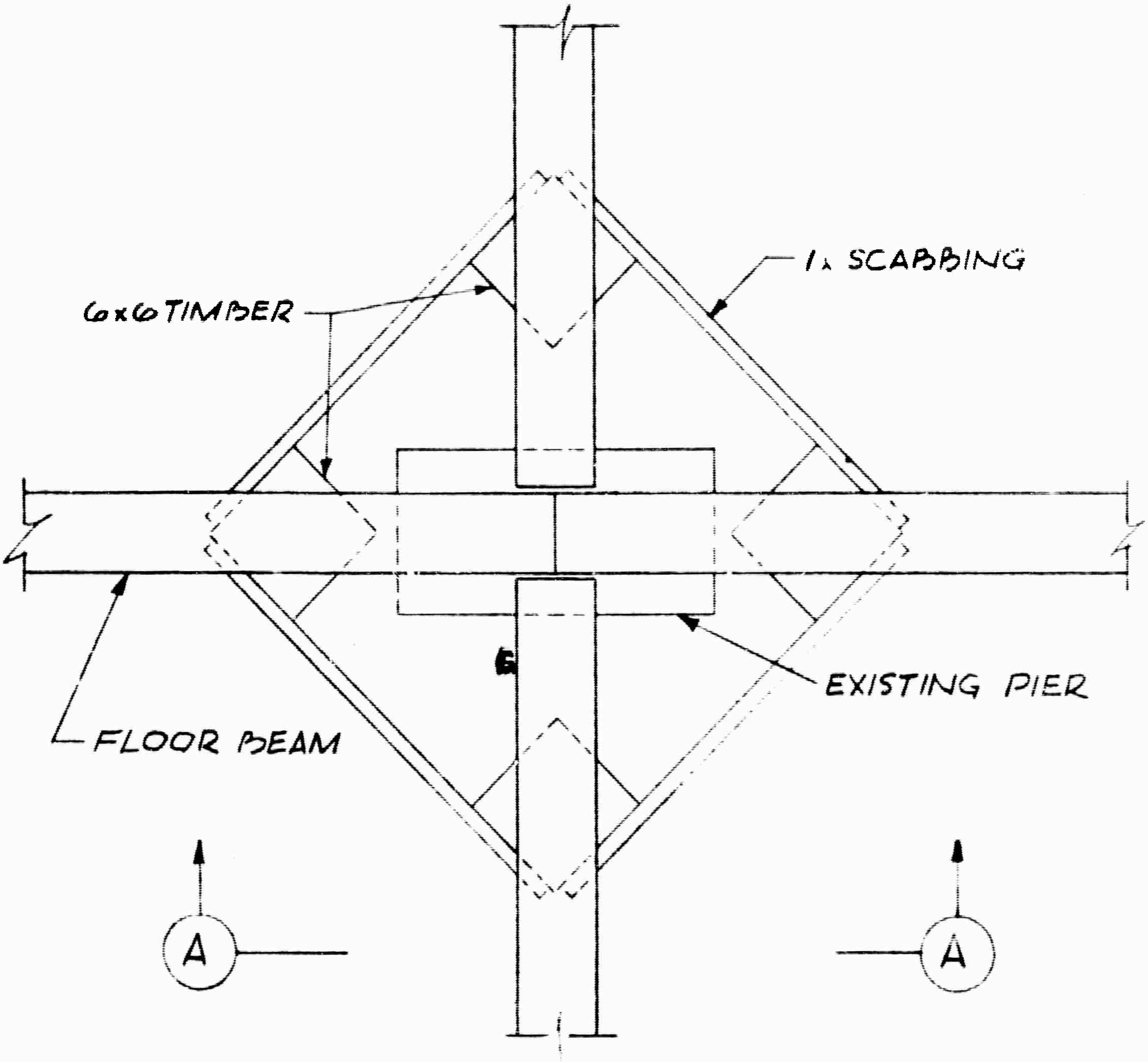
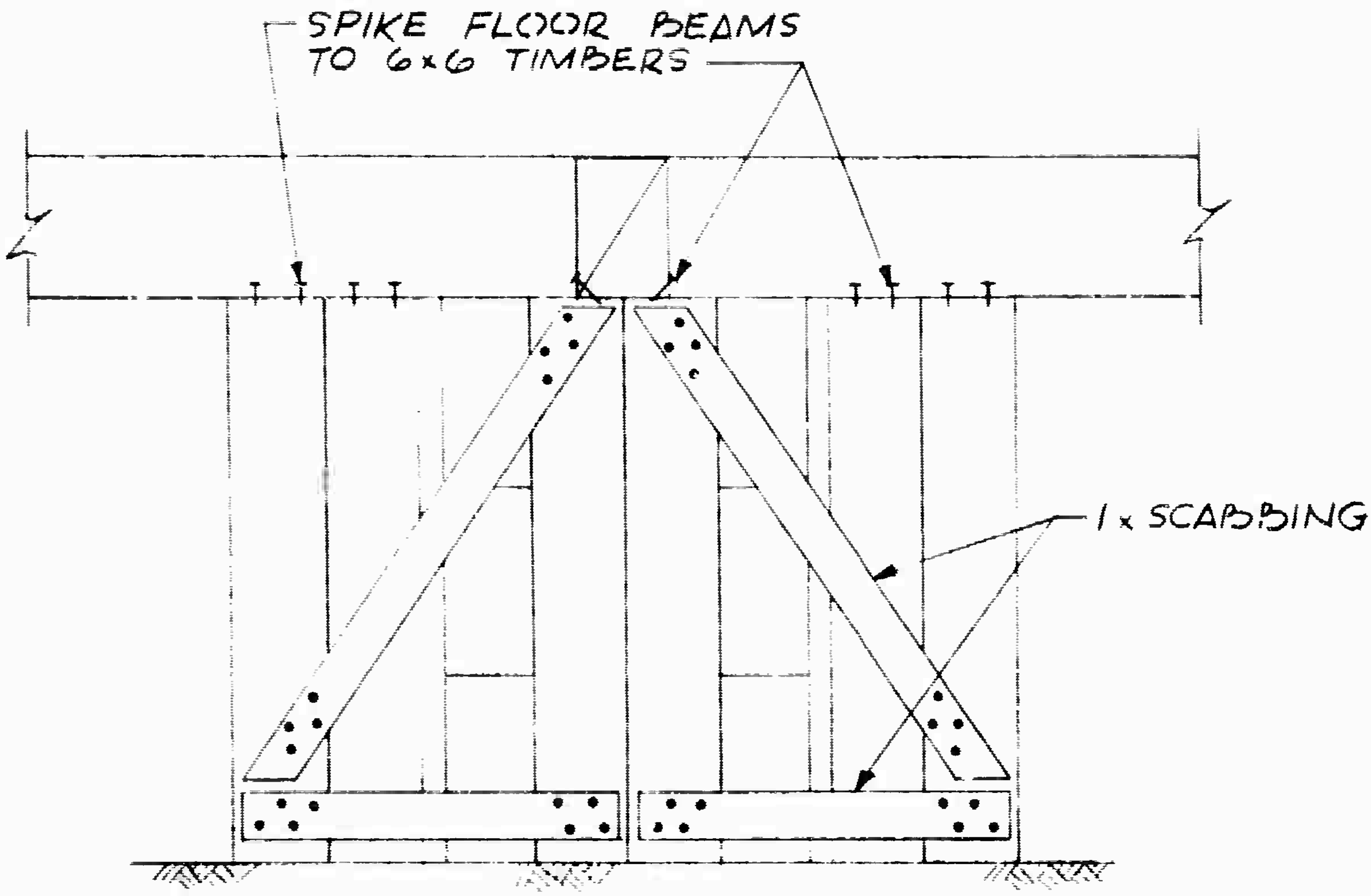


PLAN



VIEW "A-A"

Figure 25 RECOMMENDED PIER BRACING

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TECHNICAL REPORTS SCHEDULED FOR ISSUANCE BY AGENCIES PARTICIPATING IN PROJECT SHOAL

AEC REPORTS

Agency	Report No.	Project No.	Subject or Title
NBM	VUF-1001	33.2	Geological, Geophysical and Hydrological Investigations of the Sand Springs Range, Fairview Valley and Fourmile Flat, Churchill County, Nevada
SC	VUF-1002	40.5	Seismic Measurements at Sandia Stations
SC	VUF-1003	45.3	Hydrodynamic Yield Measurements
SC	VUF-1004	45.5	Device Support, Arming, Stemming and Yield Determination
SC	VUF-1005	45.6	Radiological Safety
EG&G	VUF-1006	60.4	Final Timing and Firing Report - Final Photo Report
USBM-PRC	*		Subsurface Fracturing From Shoal Nuclear Detonation
USWB	VUF-1008		Weather and Surface Radiation Prediction
USPHS	VUF-1009		Off-Site Surveillance
USBM	VUF-1010		Structural Survey of Private Mining Properties
USC&GS	VUF-1011		Seismic Safety Net
REECo	VUF-1012		On-Site Health and Safety Report

Agency	Report No.	Project No.	Subject or Title
RFB, Inc.	VUF-1013		Analysis of Shoal Data on Ground Motion and Containment
H-NSC	VUF-1014		Shoal Post-Shot Hydrologic Safety Report
H&N	VUF-1015		Pre-Shot and Post-Shot Structure Survey
H&N	VUF-1016		Test of Dribble-Type Structures
FAA	VUF-1017		Federal Aviation Agency Airspace Advisory
		DOD REPORTS	
SC	VUF-2001	1.1	Free Field Earth Motions and Spalling Measurements in Granite
SC	VUF-2002	1.2	Surface Motion Measurements Near Surface
** USC&GS	VUF-2300	1.4	Strong Motion Seismic Measurements
LPI	VUF-2600	1.6	In-Situ Stress in Granite
** STL	VUF-2400	1.7	Shock Spectrum Measurements
SRI	VUF-3001	7.5	Investigation of Visual and Photographic On-Site Techniques
SRI	VUF-3002	7.6	Local Seismic Monitoring - Vela CLOUD GAP Program

TI	VUF-3003	7.8	Surface and Subsurface Radiation Studies
USGS	VUF-3004	7.9	Physical and Chemical Effects of the Shoal Event
ITEK	VUF-3005	7.10	Airborne Spectral Reconnaissance
BR Ltd.	VUF-3006	7.15	The Mercury Method of Identification and Location of Underground Nuclear Sites
NRDL	VUF-3007	7.16	Multi-Sensor Aerial Reconnaissance of an Underground Nuclear Detonation
GIMRADA	VUF-3008	7.17	Stereophotogrammetric Techniques for On-Site Inspection
ISOTOPES	VUF-3009	7.19	Detection in Surface Air of Gaseous Radionuclides from the Shoal Underground Detonation
*** USC&GS		8.1	Microearthquake Monitoring at the Shoal Site
**** GEO-TECH		8.4	Long-Range Seismic Measurements

* This is a Technical Report to be issued as PNE-3001 which will receive TID-4500 category UC-35 Distribution "Nuclear Explosions-Peaceful Applications"

** Project Shoal results are combined with other events, therefore, this report will not be printed or distributed by DTIE

*** Report dated March 1964 has been published and distributed by USC&GS

**** Report dated December 9, 1963, DATDC Report 92, has been published and distributed by UED

LIST OF ABBREVIATIONS FOR TECHNICAL AGENCIES

BR Ltd.	Barringer Research Limited Rexdale, Ontario, Canada
EG&G	Edgerton, Germeshausen & Grier, Inc. Boston, Massachusetts Las Vegas, Nevada Santa Barbara, California
FAA	Federal Aviation Agency Los Angeles, California
GEO-TECH	Geo Technical Corporation Garland, Texas
GIMRADA	U. S. Army Geodesy, Intelligence and Mapping Research and Development Agency Fort Belvoir, Virginia
H-NSC	Hazleton-Nuclear Science Corporation Palo Alto, California
H&N, Inc.	Holmes & Narver, Inc. Los Angeles, California Las Vegas, Nevada
ISOTOPES	Isotopes, Inc. Westwood, New Jersey
ITEK	ITEK Corporation Palo Alto, California
LPI	Lucius Pitkin, Inc. New York, New York
NBM	Nevada Bureau of Mines University of Nevada, Reno, Nevada
NRDL	U. S. Naval Radiological Defense Laboratory San Francisco, California
REECo	Reynolds Electrical & Engineering Co., Inc. Las Vegas, Nevada
SC	Sandia Corporation Albuquerque, New Mexico
SRI	Stanford Research Institute Menlo Park, California

RFB, Inc.	R. F. Beers, Inc. Alexandria, Va.
STL	Space Technology Laboratories, Inc. Redondo Beach Park, California
TI	Texas Instruments, Inc. Dallas, Texas
USBM	U. S. Bureau of Mines Washington, 25, D. C.
USBM-PRC	U. S. Bureau of Mines Bartlesville Petroleum Research Center Bartlesville, Oklahoma
USC&GS	U. S. Coast and Geodetic Survey Las Vegas, Nevada
USGS	U. S. Geologic Survey Denver, Colorado
USPHS	U. S. Public Health Service Las Vegas, Nevada
USWB	U. S. Weather Bureau Las Vegas, Nevada